Case study – seismic design and soil-structure interaction of a critical wharf facility in soft soils

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ABSTRACT

A case study of performance based seismic design (PBD) for a critical wharf facility is presented and compared to conventional design methods. The Oil Wharf at the Lyttelton Port is a critical infrastructure link for New Zealand's South Island. The wharf suffered extensive damage due to the Canterbury earthquakes and is to be replaced with a new structure. The design of a replacement wharf utilised PBD principles throughout every aspect of the project, including probabilistic seismic hazard assessment, selection of design ground motions, nonlinear site response analyses and nonlinear, dynamic deformation and soil structure interaction analyses.

The site is located on soft marine sediments only four kilometres from the epicentre of the M6.3 22 February 2011 Christchurch earthquake. A unique strong motion dataset is available at the site; a seismograph on soft soil is located 100m away and another on rock is located 900m away. Site response analysis using the recorded rock ground motions as input was able to match the recorded soft soil ground motions over a range of earthquake sizes. Site response analyses were then used to derive the design spectra for structural design.

Nonlinear deformation analyses (NDAs) were able to replicate key displacement mechanisms observed in the Canterbury earthquakes. Predicted displacements matched the surveyed slope deformation data which enabled real world validation of the analyses results. A series of NDAs were performed to design the new wharf structure. Performance based design resulted in significant construction cost savings for the design of the new wharf structure when compared to conventional design methods.

1 INTRODUCTION

Lyttelton Port of Christchurch (LPC) has experienced many large earthquakes and thousands of aftershocks during the 2010-2011 Canterbury Earthquake sequence. Consequently, LPC and their consultants and contractors have a very clear understanding of Performance Based Design as they have observed the actual performance of their land and structures through a wide range of earthquake magnitudes.

An example of one of their damaged structures is the Oil Berth, which is a critical facility that supplies oil, petrol, LPG and bitumen to New Zealand's South Island. The Oil Berth suffered extensive damage in the 2010-2011 Canterbury Earthquake sequence.

A replacement wharf has been designed which will replace the existing wharf. This paper describes the performance-based design of the replacement wharf, including the following key aspects:

- Utilising the extensive post-earthquake observations (including survey measurements, structural inspections, strong motion data) to benchmark geotechnical analysis and design
- Design to the performance based ASCE 61-14 code for the Seismic Design for Piers and Wharves using advanced methods for site response analysis, seismic hazard assessment and dynamic non-linear deformation and soil structure interaction analysis

A comparison between the performance based design approach and conventional design is presented.

2 LYTTELTON PORT OIL BERTH

The Oil Berth is located in Lyttelton Harbour, New Zealand. The existing wharf structure comprises a wharf supported on timber piles, pipelines, a reinforced concrete seawall, mooring bollards and thrust blocks. Fender blocks are supported on four 600mm diameter steel encased concrete piles.

A new oil wharf, 63.3m long by 15m wide is proposed. The proposed wharf has a 0.8m thick reinforced concrete deck and is supported on 39 no. 1.2m diameter steel encased concrete piles. A 36.5m long by 6m wide access jetty will link the wharf with the shore, which will incorporate a 0.8m thick reinforced concrete deck.

The site is situated on reclaimed land that overlies harbour deposits consisting of soft to stiff silts with medium dense to very dense sands from approximately 40 m depth and volcanic boulders and rock at approximately 50 m depth. The reclamation process involved construction of a breakwater by end dumping quarried basalt gravel on top of the natural marine deposits. Progressive slope failures were reported during construction. Reclamation of land behind the breakwater was then carried out with hydraulic fill dredged from the harbour bed.

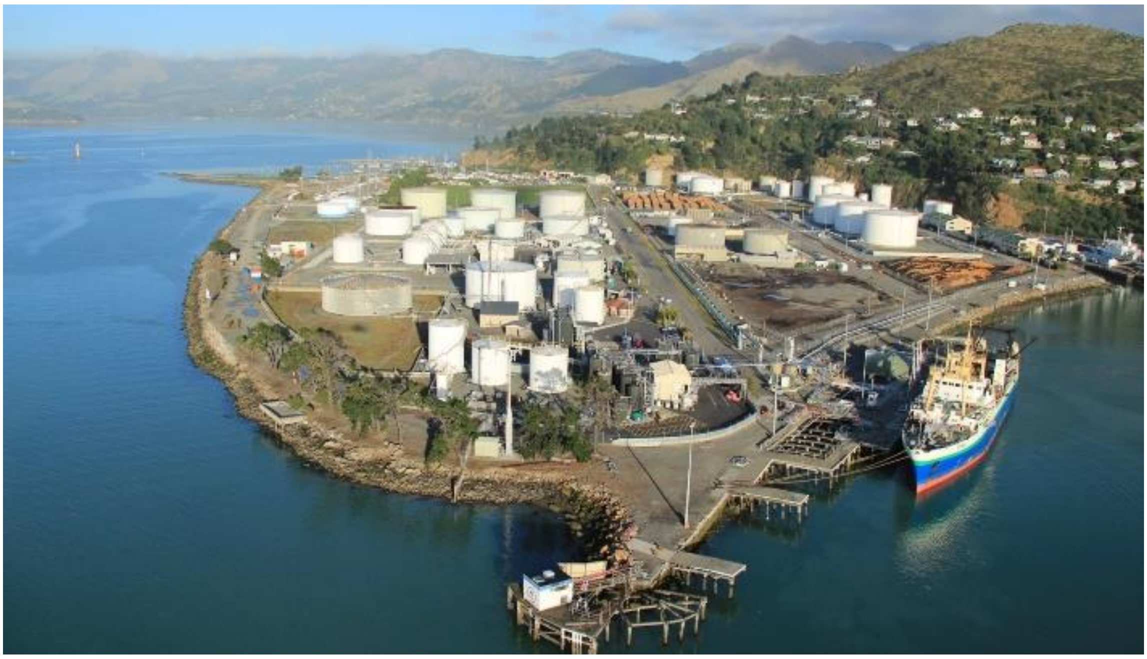

Figure 1: Lyttelton Port showing a vessel moored at the existing Oil Berth

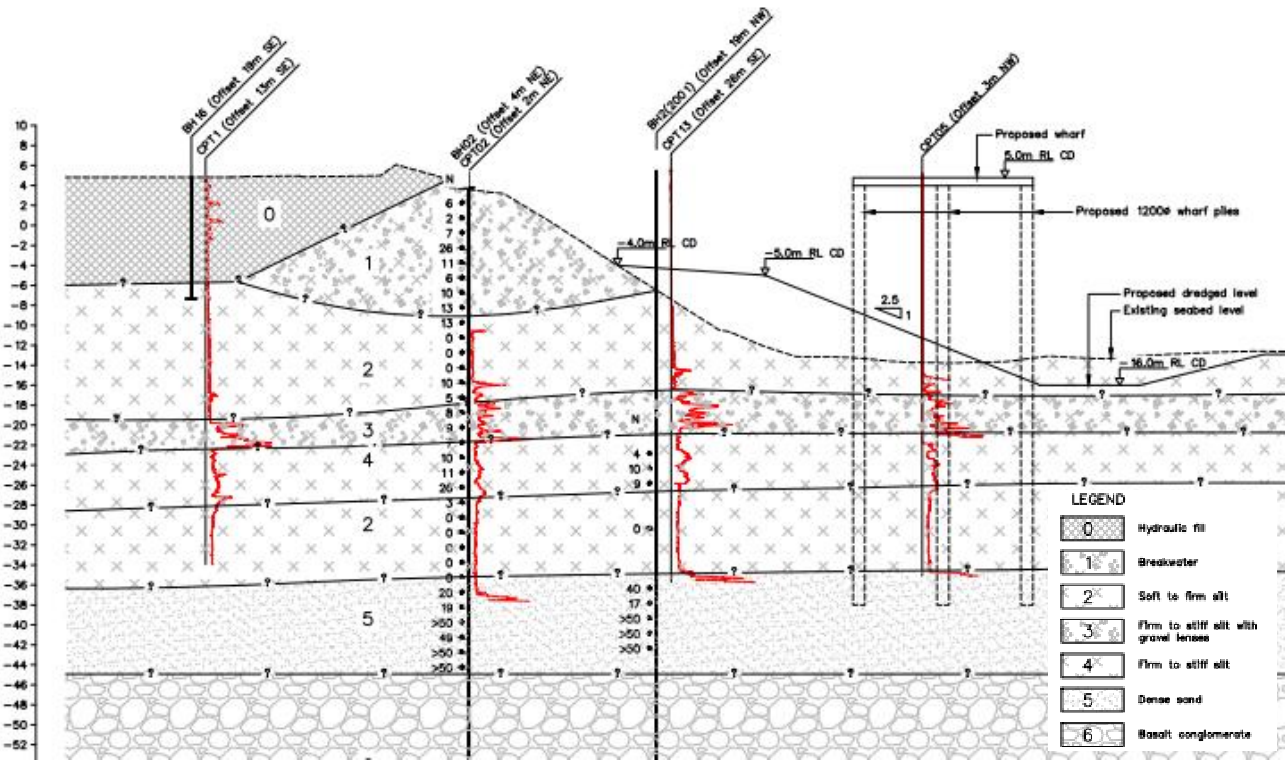

Figure 2: Geotechnical cross section through proposed Oil Berth

3 DESIGN METHODOLOGY

3.1 Performance requirements

The replacement Oil Berth is designated as a 'High' Design Classification in terms of the ASCE Standard 61-14 Seismic Design of Piers and Wharves. This is because the Oil Berth is deemed essential to the region's economy and post-event recovery. As such the wharf and associated breakwater slope must be designed to provide the following performance requirements:

- Long term static factor of safety greater than 1.5
- Post- earthquake static factor of safety greater than 1.1
- Minimal damage in a 75 year return period earthquake
- Controlled and repairable damage in a 475 year return period earthquake
- Life safety protection in a 2500 year return period earthquake

At the Oil Berth site there are many geotechnical challenges in achieving these requirements, including weak natural soils, potentially variable fill material, poor existing slope performance and an increase in slope height through dredging. In addition the existing Oil Berth must be kept operational during construction.

3.2 Design outline

Four key steps were carried out in the geotechnical design of the Oil Berth:

1. Site specific seismic hazard assessment. This was completed using a Probabilistic Seismic Hazard Analysis (PSHA). The main motivations for carrying out the PSHA were to better quantify the aftershock hazard following the 2010-2011 Canterbury Earthquakes and to better characterise the 2500 year return period event. The PSHA resulting in a significant reduction in seismic hazard compared to code values.

Table 1: Seismic design cases as per ASCE 61-14

Design case	Ground motion probability of exceedance	Peak Horizontal Ground Surface Acceleration (PGA_H)		Required performance level
		NZS 1170.5 (Class D)	Site-specific seismic hazard assessment	
OLE Operating level earthquake	50% in 50 years (72-year return period)	0.18g	0.153g	Minimal damage
CLE Contingency level earthquake	10% in 50 years (475- year return period)	0.35g	0.330g	Controlled and repairable damage
DE Design earthquake	2% in 50 years (2500- year return period)	0.63g	0.500g	Life safety protection

2. Ground motion selection using the Generalised Conditional Intensity Measure (GCIM) approach. Ground motion selection and analysis was desired to take advantage of the recorded rock and soft soil time histories available from the Canterbury earthquakes. The selected ground motions are summarised in Table 2. Both the East-West and North-South components of the ground motions were used.
3. Non-linear site response analysis to determine the appropriate design spectra for seismic assessment of the wharf structure. This step was undertaken at this site because the site investigation results indicated Site Subsoil Class E in terms of NZS1170.5 and Site Class F in terms of ASCE 7 (2005). Site response analysis therefore enabled a reduction in seismic demand from NZS 1170.5 and met the requirements of ASCE 61-14 which requires a site response analysis for Site Class F sites.
4. Dynamic non-linear deformation analysis (NDA) was undertaken to assess lateral loadings on the piles, soil-structure interaction and the pile pinning effects on the breakwater slope. Inertial loading from the wharf superstructure and kinematic loading caused by lateral ground deformations was considered simultaneously.

The third and fourth tasks are described in more detail in the following sections.

4 NON-LINEAR SITE RESPONSE ANALYSIS

The dynamic response of soil deposits beneath a site has a significant influence on the ground motion hazard of engineered structures. During seismic shaking, bedrock motions have the potential to be both amplified and/or damped when travelling through surficial soils, ultimately affecting the resulting seismic forces which are imparted on structures at the surface. These effects depend on the ground profile, the physical characteristics of each soil layer and the strength and direction of shaking.

1D site response analyses were undertaken using a non-linear method (DMOD2000). The analysis methodology was verified through back analyses of the recorded rock ground motions at GNS monitoring station LPCC. The input from the calibration model was able to match the recorded soft soil ground motions at GNS seismograph station LPOC over a range of earthquake sizes.

Equivalent linear methods (e.g. SHAKE2000) were unable to capture the response of the soft soils to strong ground shaking.

Table 2: Summary of selected seismic ground motions

Ground motion source	Earthquake magnitude (Mw)	Distance of record source from fault rupture (km)	Ground Vs30 at record source (m/s)
LPCC 4/9/10	7.1	22.1	780
LPCC 22/2/11	6.2	7.1	780
LPCC 13/6/11	6.0	5.8	780
LPCC 23/12/11	5.9	12.4	780
Chi Chi, Taiwan 1999 TCU076	6.2	14.7	615
Kobe, Japan 1995 Nishi-Akashi	6.9	7.1	609
Kocaeli, Turkey 1999 Gebze	7.5	10.9	792

Once the back analysis could demonstrate satisfactory results forward analysis was undertaken to verify that the PSHA spectra for surface soils was appropriate given the soft soils at the site.

The assessed site specific response spectra have been calculated as the mean spectra from a suite of 28 ground motion analyses (based on the seven earthquake records which are summarised in Table 3). Figure 2 shows the spectral accelerations from the site response analysis compared to the PSHA spectra curves for the CLE design events.

The main conclusions from the site response analysis are:

1 The natural soil period is approximately 0.7 to 1.3 seconds depending on the level of shaking, as evaluated by comparing the ratio of ground response spectra to rock outcrop spectra. Given that the period of the wharf structure is estimated to be around 2.0 seconds we consider that the use of the PSHA spectra curve in Figure 2 is appropriate for the site.

2 For long spectral periods (T>~1s) the site response analysis indicates a spectra significantly less than the PSHA curves using Vs30=180m/s. However, there may be physical processes at long periods (i.e. surface waves, topography or basin effects) that are present at the site but unable to be captured within the 1D analysis. Therefore it was considered that using the lower spectral acceleration values from the site response analysis (as shown in Figure 3) was not appropriate for structural design.

5 DYNAMIC TIME HISTORY ANALYSIS

The analysis methodology used to assess the slope stability and evaluate the seismic slope displacements comprised three steps:

1. The model parameters were evaluated using correlations with the site investigation data.
2. Back analysis of the land performance which was observed during the Canterbury earthquakes.
3. Dynamic forward analysis using the ground motions presented in Table 2 scaled to the appropriate OLE, CLE and DE events.

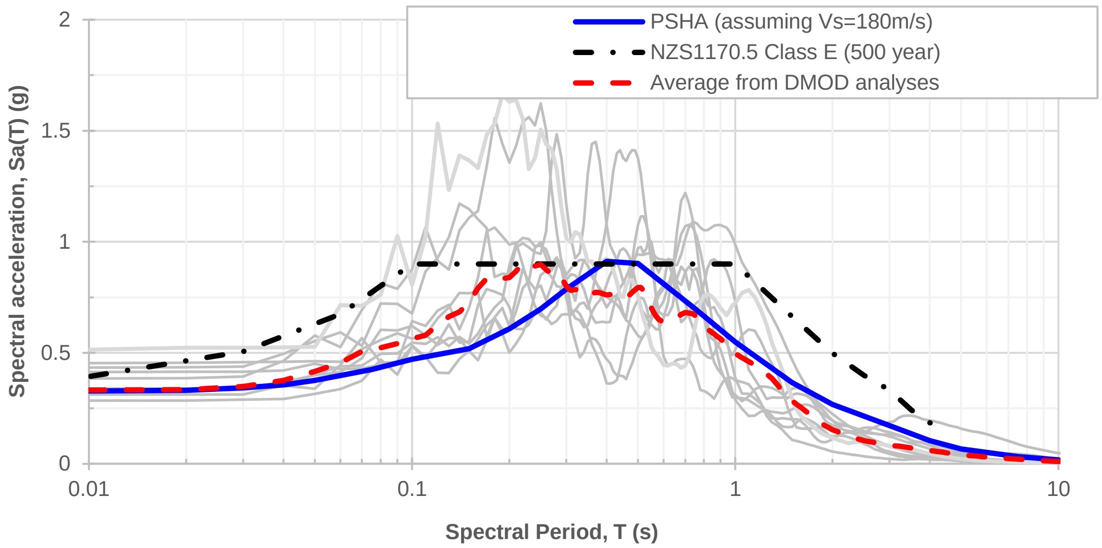

Figure 3: Response spectra plot for the CLE event showing the results from the PSHA (blue line), code spectra from NZS1170.5 (black line) and average spectra from the DMOD analyses (red line). The light grey lines are the spectra from individual DMOD analyses.

5.1 Geotechnical model

The harbour deposits were subdivided into five layers, with a further division in 'seaward' and 'landward' soil layers to account for the difference in strength due to consolidation of the landward soils under the weight of the hydraulic fill and breakwater.

The undrained shear strength profiles for the harbour deposits were derived from the CPT results using an Nk factor equal to 12. Nk = 12 was used based on correlation with shear vane testing.

In the dynamic model, the HS Small constitutive model was used for all materials, except the hydraulic fill, dense sand and rock layers, to account for stiffness degradation under large strains and to incorporate hysteric damping effects. The HS small model stiffness and damping parameters were derived using a combination of shear wave velocity data, CPT results, machine-drilled borehole data, and triaxial and oedometer laboratory testing of harbour deposits at Cashin Quay.

5.2 Back analysis

A back analysis of the slope was undertaken using the slope movement data which was collected following the Canterbury earthquakes. The purpose of the back analysis was to validate the PLAXIS slope stability model and geotechnical parameters.

The back analysis and calibration of the dynamic PLAXIS model was performed using the earthquake record from the GNS LPCC seismograph station was used as a bedrock motion. The magnitude and distribution of vertical and horizontal seismic displacements predicted by the model were compared to post earthquake survey measurements.

Table 4 summarise the results from the dynamic analysis. The model was able to provide a reasonable match with the model displacements within ten percent of the observed displacements. This provided confidence in the methodology to enable forward analysis incorporating the revised slope profile and the addition of the wharf piles.

Table 3: Back analysis of seawall displacement during the Canterbury earthquakes – comparison between measured values and dynamic PLAXIS model results

Parameter	4 September 2010 earthquake	22 February 2011 earthquake
Measured horizontal displacement at seawall	0.26m	0.4m
Horizontal displacement from dynamic PLAXIS model at seawall	0.23m	0.37m

5.3 Forward analysis

The earthquake records presented in Table 3 were used as a bedrock motion in a dynamic PLAXIS model. A total of 42 dynamic analyses were undertaken, using both the North-South and East-West components of all seven earthquake records scaled to the OLE, CLE and DE levels of shaking. The model was used to directly estimate the magnitude and distribution of vertical and horizontal seismic displacements.

Figure 6 shows the wharf horizontal displacements for the CLE design case. The largest cyclic displacements were observed in the Kocaeli EW and NS records which included significant forward directivity effects. ASCE 61-14 allows that the mean value be used for design if more than seven time history records have been used. The mean maximum horizontal wharf displacement from all the analyses was 0.21m and the mean displacement at the end of the shaking was 0.14m. These values were used in design of the wharf piles.

5.4 Comparison with conventional pseudo-static analysis

A conventional analysis was also completed with a pseudo static PLAXIS analysis to determine the yield acceleration of the slope. The yield acceleration, as well as the peak ground accelerations from the PSHA were then used as inputs into a sliding block analysis (Jibson 2007) to estimate the peak horizontal seismic slope displacements. This method indicated the yield acceleration of the proposed slope (including fill buttress) to be approximately ac = 0.090, which resulted in an estimated displacements much larger than the dynamic analysis. Table 5 provides a comparison between the estimated wharf displacements between the pseudo-static and dynamic analyses.

The magnitude of slope displacement has a direct influence of the soil loading on the piles. Prior to the dynamic analysis being undertaken the pile casings needed to have a steel thickness of 24mm to meet the requirements of ASCE 61-14. Through adopting the slope displacement values from the dynamic analysis the pile casing thickness was able to be reduced to 16mm while still meeting the ASCE6 61-14 requirements. This resulted in a project cost saving of NZD$600,000.

Table 4: Comparison between dynamic and pseudo-static analysis

Design case	Mean maximum horizontal wharf displacement		Percentage increase from dynamic to pseudo static
	Dynamic analysis	Pseudo static analysis	
CLE	0.21m	0.27m	30%
DE	0.44m	0.66m	50%

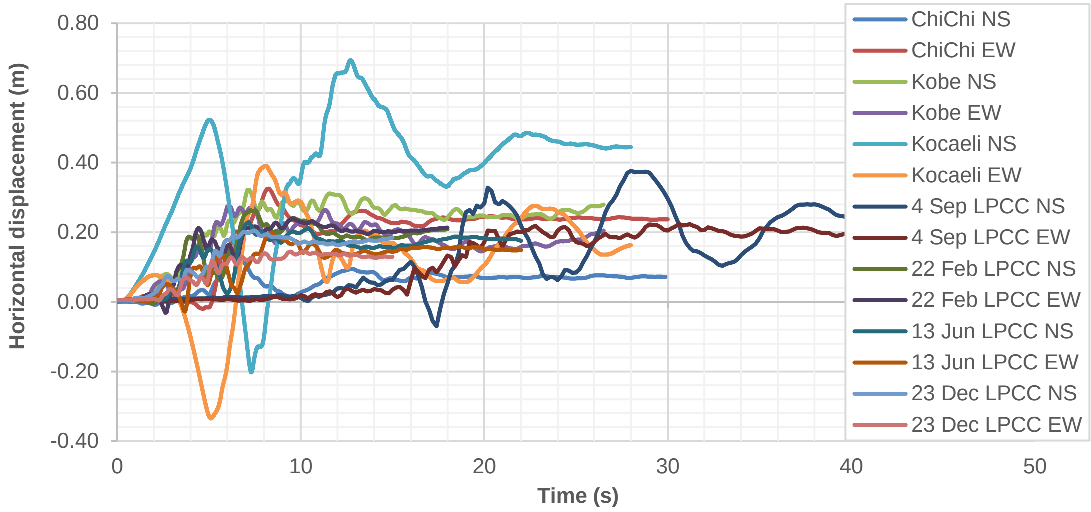

Figure 4: Wharf horizontal displacement versus time plots from the PLAXIS dynamic analysis for all fourteen earthquake cases scaled to the Contingency Level Earthquake (475 year return period)

6 CONCLUSIONS

This paper presents a case study where advanced analysis was undertaken in every step of the project; (1) characterisation of seismic hazard, (2) selection of ground motions for analysis, (3) assessment of the dynamic soil response at the site, and (4) analysis of soil-structure interaction. Advanced analyses were suited to this particular project due to the soft soils at the site and the high seismic design standard required for the proposed wharf.

The advanced analysis resulted in significant reductions in seismic actions on the wharf structure. This resulted in significant reductions in construction cost which were well in excess of the cost of the analysis.

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